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Geotechnical Investigation and 2D Electrical Resistivity Survey of a Pavement Failure in Ogbagi Road, Southwestern Nigeria

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Abstract

This paper presents the result of a geotechnical investigation and 2D electrical resistivity survey carried out along Ogbagi-Ese Akoko road in Ondo State, Southwest Nigeria. The lithology of the study area is comprises of Granite, Charnockite, Quartzite and Gneiss. The causes of highway pavement failure were delineated using geotechnical and resistivity dipole-dipole array method. Soil samples were taken for geotechnical testing at 1.0m depth and a survey with horizontal profiling having an electrode separation of 5.0m over a traverse of 300m were carried out using resistivity method. The natural moisture content, specific gravity, consistency limit, plasticity index, linear shrinkage, sand, fines, and clay ranges from 18.56-11.3%, 2.683-2.761, 44.8-33.1%, 14.6-20.55, 9.6-11%, 40-52%, 30.4-58.1%, 37.4-23.7% respectively. Along the failed segments, low resistivity, weathered, water-absorbing substratum and linear features suspected to be fracture zones and joints were delineated using resistivity survey. The results show that the clayey grade soil below the highway pavement and identified suspected structural features are the major geologic factors responsible for the poor drainage conditions and the highway pavement failure.

Keywords: Geotechnical Investigation, 2D Electrical Resistivity, Highway Pavement Failure, Southwestern Nigeria.

Introduction

Major Nigerian roads are known to fail shortly after construction and well before their design age (Olorunfemi et al. 1987). The inadequate provision of drainage facilities to drain out or remove excess water quickly on or in the road pavement after heavy showers leads to failure of pavements. Rainfall is the most damaging environmental factors on pavement in tropical Africa. The road network in Nigeria is experiencing a systematic deterioration equivalent to an asset loss of about N80 billion per annum (Oguara, 2001). Rainfall as well as Poor construction materials, bad design, usage factor, poor drainage network are some of the factors considered as responsible for these failures.

Geological factors are rarely considered as precipitators of road failure even though the

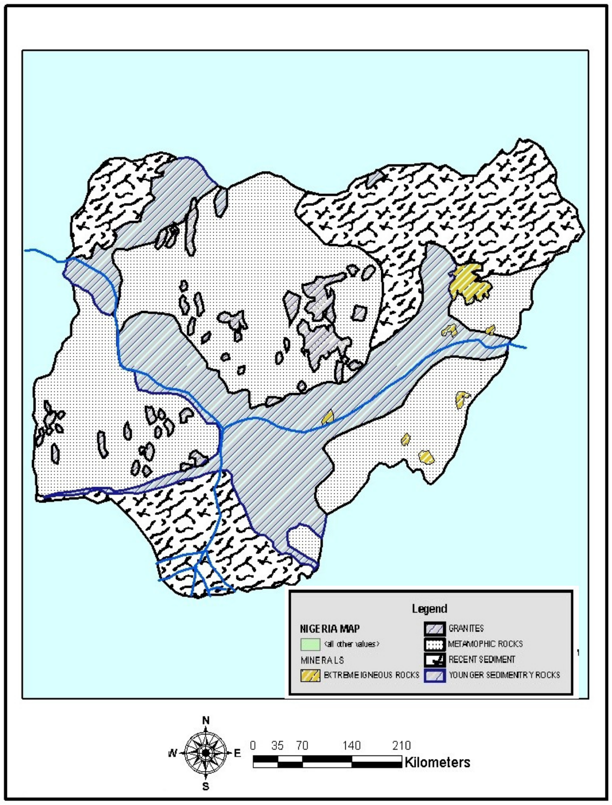

Fig. 1. Geological Map of Nigeria showing Ondo state (Ajibade et al., 1989)

highway pavement is founded on the geology (Olorunfemi et al. 1987). This is due to non-appreciation of the fact that proper design of highway requires adequate knowledge of subsurface conditions beneath the highway route. The non-recognition of this fact has led to loss of integrity of many highway routes and other engineering structures across the country (Olorunfemi et al. 1987). It is therefore vital for engineers to carry out pre-design investigations of engineering sites.

In view of the efforts of the Federal Road Maintenance Agency (FERMA) to rehabilitate failed segments of the roads across the country, it is imperative that adequate consideration is given to the causes of the failures so as to ensure that sufficient safeguards are incorporated in their subsequent rehabilitation.

Road failures could be defined as a discontinuity in road network resulting in cracks, potholes, bulges and depressions (Aigbedion, 2007). Failure of road pavement can occur in the form of pitting and rutting, waviness was adjudged the most common form of road failure (Gidigasu, 1974; Adeyemi and Oyeyemi, 2000). The majority of road failure in the tropics can be attributed to geotechnical factors as reported workers such as (Gidigasu, 1972). (Poor construction materials, bad design, poor drainage network as some of the factors responsible for road failures (Meshida, 1985; Adeyemi, 1990; Momoh et. al, 2008).

This problem of premature failure of our roads has been of great concern to most of the geotechnical engineers. It is important to note that in many cases, the materials on which these roads were constructed were not in harmony with the road sub-grade specifications that in some cases may be good enough; making Nigeria roads to fail before the life span elapsed. Improper drainage network have led to the occurrence of potholes, thus resulting to road failure.

De Graft Johnson (1972) proposed a criterion for rating a probable field performance of lateritic soil and position in pavement based upon aggregate as well as specific gravity and water absorption tests. Cement stabilization for lateritic soil building or walling and lime/bitumen stabilization for road construction in temperate soils (Ola, 1983). Aguda (1982) suggested that inadequate provision of drainage system in Oyo State causes pavement failure.

Method of sample penetration, firing, degree of laterization (Sesquioxide Coatings) as estimated from index test and correlated with the crushing strength of fires bricks such that the degree of laterization of clay controleed by strength of bricks (Adeyemi, 1990). Significant differences need not exist between geotechnical properties of soil below

stable zones and unstable sections of flexible highway pavement in the tropics. Depreciation in the shear strengths and compressibility of soil upon inundations (by carrying out shear strengths and consolidation tests) do reduce the bearing capacity, leading to foundation failures. The reduction in cohesion is as much as between 28kpa to 437kpa while the angle of internal friction decreased from 21 to 34 to 19 and 31. The influence of cement on the compaction characteristics of soil increases with the energy of compaction (Adeyemi and Oyeyemi, 2000; Ogunsawu, 2000; Adeyemi et al., 2003).

The present work attempts to investigate the geological factors in terms of the nature of the subsoil, the near surface structures and the bed rock structural disposition as possible causes of failures along the Ogbagi – Ese Akoko road using geotechnical and resistivity methods.

Regional and Site Geology

Figures 1-4 show the location of the study area. The geographic coordinates of the study area are longitude 05° 45[1] E to 05° 49[1] E and latitude 07° 33[1] N to 07° 37[1] N. The study area is through Ikare-Ado-ekiti Federal trunk B and via through 10km to Ogbagi-Ese road, some roads linking one street to the area and the existing footpaths.

The study area lies within the basement complex of the South-Western Nigeria. The rocks within the study area are: Older granites, gneiss, Granite gneiss, Charnokitic gneiss Garnitiferous gneiss, Charnokite, grey gneiss and they are believed to have evolved in at least four orogenic events namely: the Pan African (600±150My), The Kibaran (1100±200My),The eburnean (2000±My) and the Liberian (2800±200My). The Migmatite-gneiss complex dominate the basement complex in the study area composed of fairly uniform biotite and biotite – hornblende-gneiss with locally intercalated bands of aplitic quartz veins (Ajibade and Wright, 1980).

The climate of study area is of Tropical Rain Forest with mid monthly temperature and their ranges are about 30°C and 36°C respectively. The mean monthly humidity is less than 70% (Elkanade, 2000). The drainage pattern observed in this area is a dendritic drainage pattern.

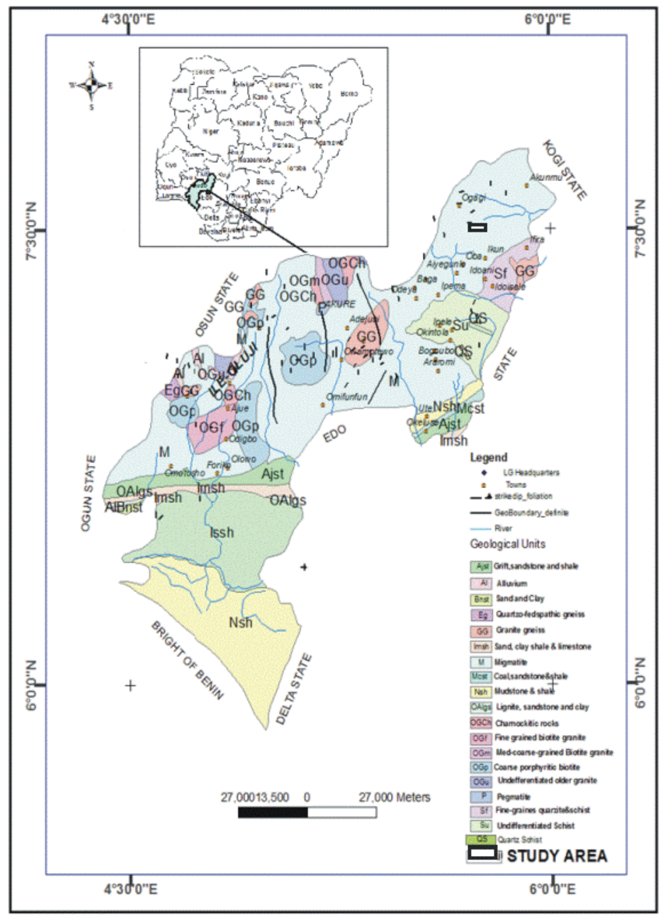

Fig. 2. Geological map of Ondo State showing the study area (After Nigeria Geological Survey Agency.)

Materials and Methods

Geophysical survey

The electrical resistivity imaging data were acquired using ABEM SAS 1000 terra meter along Three

traverses with Dipole-dipole array using a multi electrode system The equipment is capable of measuring apparent resistivity with induced polarization (IP) or self Potential (SP) at the same time, though with increase data acquisition time. The electrode spacing of 5m and total length covered was 300 m. The traverse was acquired in the North-South direction (A-B, C-D and E-F) along the three study road segments. The control stable and failed segments are 100m. The data is inverted from apparent resistivity through "true" resistivity by Earth-Imager software "Diprowin software".

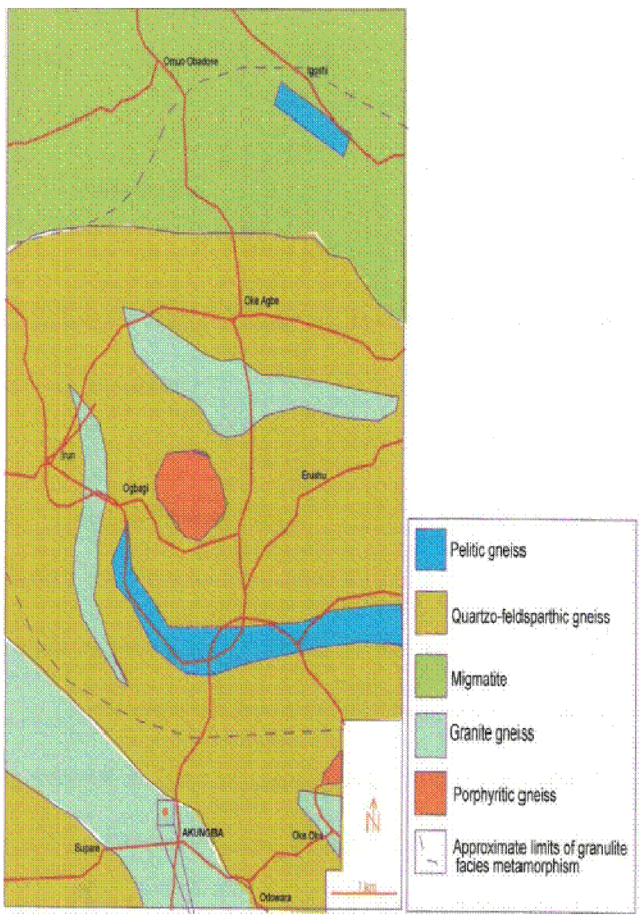

Fig. 3. Geological map of Akoko area (Rahaman and Ocan, 1978)

The goal is to create an image of the ground in terms of electrical resistivity showing both the lateral and depth extent of area of investigation. The resistivities of the blocks were iterated and adjusted until the calculated and field apparent resistivities agreed to barest minimum differences (Loke, 2004). The Pseudo section obtained from DIPROWIN SOFTWARE using Jacobian iteration is presented below. Eight iterations was done with the RMS error as 2485076E+01, 1547066E+01, 116306501E+01, 2079949E+01, 5909228E+00 from both Langranghian and Jacobian and topographic correction is applied.

Fig. 4. Failed section of Ogbagi-Ese Akoko road.

Geotechnical analysis

The samples for this project were collected from roadsides along Ogbagi-Ese road, which involve digging ten pits to depth between 0.62m to 1.2m. The soil samples were major collected from damaged sections of the road as well as un-damaged sections to serves as control. Diggers, cutlass and hoe was used to dig the ground and to collect soil samples, fresh samples were collected and immediately wrapped in sample bag to minimize loss of moisture. To enhance accurate result, impediment such as organic soils decayed debris, plant residues and animal remains were removed from the samples.

Sample preparation

The sample collected were air-dried for weeks before being subjected to laboratory test except those of moisture contents which were immediately carried out in the laboratory. The samples were stirred at regular intervals during the period of air-drying. After air-dried, part of the soil were sieved through 425umbs sieves. The un-sieved soil was, however, used for Atterberg limit and other tests.

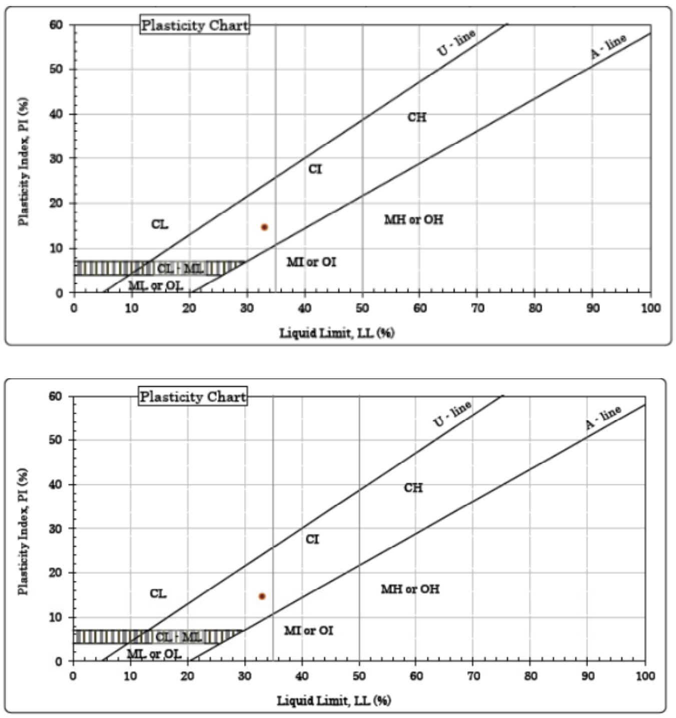

Fig. 5. The plasticity chart from the geotechnical result (CI- Silt clay of low compressibility, CH- Silt of low plasticity, MI- Silt of low plasticity, CL- Clay, MH- Clay of high plasticity, ML- Clay of low plasticity).

Laboratory test

The laboratory test carried out for this project is divided into three, they are:

The classification (Index properties) Tests

The engineering standard (AASHTO level) Tests

The California Bearing Ratio (CBR) Tests

The American Society for Testing and Materials (ASTM) standard was utilized in this laboratory test. The first test can be regarded as the preliminary test while other two can also be called or regarded as semi-empirical test.

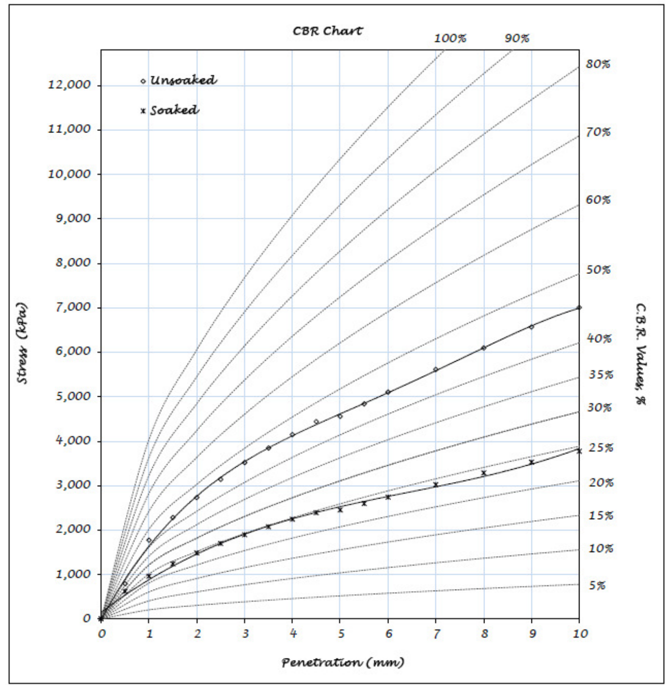

Fig. 6. The CBR chart of sample 1.

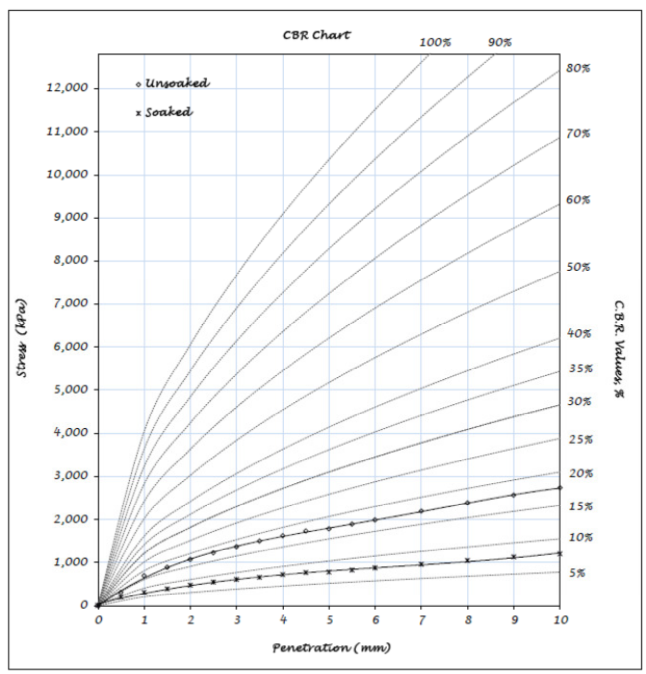

Fig. 7. CBR chart of sample 2.

Results and Discussion

The results of the laboratory tests includes natural moisture content, specific gravity, grain-size distribution curves, Atterberg limits, compaction test and California Bearing Ratio (CBR) Tests are shown in Tables 1-4 respectively.

Table 1. The result of the grain size test

Soil type	Sample 1	Sample 2
Gravel	1.9%	8.8%
Coarse Sand	3.4%	6.7%
Medium Sand	17.3%	23.4%
Fine Sand	19.3%	22.7%
Total Sand Content	40.0%	52.8%
Fines	58.1%	38.4%
Silt	20.7%	15.0%
Clay	37.4%	23.4%
Hydroscopic Moisture		
Content %	6.2%	4.5%
Bulk Weight	500.0g	500.0g
Dry Weight	470.8g	478.5g

The natural moisture content of the soil ranges from 18.56% to 11.3%. The high natural moisture content observed in the study area is due to poor drainage networks (Table 2).

The liquid limits ranges from 44.8% to 33.1%. While the corresponding plastic limits ranges from which results in plasticity index that range from18.6% to 24.3% which result in plasticity index that range from 14.6 to 20.55 (Table 4).

The value of liquids limits are generally higher than the maximum value of 30% stipulated by the federal ministry of work for high way sub-base soils, the value of plastic limits are generally lower than the maximum value of 25.00 recommended. The high plastic limit is due to poor drainage networks. The values of plasticity index are higher than maximum value of 12.00 recommended based on French recommendation of road and tropical Africa. Since the plastic limits are less than 25.00, the soils would exhibit low to medium swelling potential (Ola, 1983).

According to Casagrande Chart classification, the soil samples from the studied area fall above the A-line indicating predominantly inorganic soils. The soil samples plot in the medium plasticity area. This implies that the soil samples in the studied area are moderately plastic soil. These may be due to quartz and mica present in the parent rocks (Gidigasu, 1974).

Madedor (1983) recommended that linear shrinkage should not be more than 8% for sub-base and less or equal to 10 for sub grade. Thereby making the unsuitable as sub-grade and posing field compaction problem. The linear shrinkage from the sample is 9.6 and 11.0 indicating good sub-grade; therefore, the failed section in this area may be due to poor drainage system.

Montmorillonite has a high plasticity index. The soils are active soils, which are poor for road construction. Soils with Montmorillonite are known for their high propensity to swell in the presence of water (Stephenson, 2004).

The maximum dry density index (MDD) and optimum moisture content (OMC) values are 2012kg/m^3 and 12.5% to 18.8% for the studied area (Table 2). The presence of silty and clayey sand in these samples is responsible for their low MDD. In addition, the OMC of this sample is due to the presence of absorbed water surrounding the clay particles. The MDD of the soil samples are less than the recommended value of 2165kg/m^3 for Nigeria soils. Therefore, the soils are poor sub-grade materials.

Table 2. The result of the linear shrinkage and moisture content test

Linear shrinkage		**Moisture content test**			**USCS group symbol: (CL)**	
		TEST No	1	2	Plasticity Index	20.55
Original Length L_0 (mm)	140 Container No	A	B			
		Wet Soil Container g	26.48	28.95	Consistency index	1.28
Final Length L (mm)	128	Dry Soil Container g	24.40	26.70		
		Container Empty g	13.20	14.40	Liquidity Index	0.28
Linear Shrinkage	8.6	Dry Soil g	11.20	12.30		
(1.(L/L$_2$)*100		Loss of water g	2.08	2.25	Flow Index	6.59
Shrinkage limit =	9.6	Moisture Content %	18.60	18.30		
		Moisture Content =	18.5		Swell Index 0.41	
Linear shrinkage		**Moisture content test**			**USCS group symbol: (CL)**	
		Test No	1	2	Plasticity Index	14.60
Original Length L0 (mm)		Container No	A	B		
		Wet Soil & Container g	25.65	28.10	Consistency Index	1.49
Final Leght L (mm)		Dry Soil & container g	24.40	26.70		
		Container Empty g	13.20	14.40	Liquidity Index	0.49
Linear shrinkage =	6.4	Dry Soil g	11.20	12.30		
(1.(l/l3)1*100		Loss of Water g	1.25	1.40	Floe Index	5.59
Shrinkage Limit =	11.0	Moisture Content	11.20	11.35		
		Moisture Content =	11.3		Swell Index	0.34

Table 3. The liquid limit test result

Liquid Limit (LL) Test			LL= 44.8%	
Test No.	1	2	3	4
Container No.	A	B	C	D
Wet soil & Container g	34.64	34.20	33.54	34.67
Dry soil &container g	28.60	28.10	28.70	27.60
Container Empty g	14.40	14.20	13.60	13.00
Dry Soil g	14.20	13.90	15.10	14.60
Loss of Water g	6.04	6.10	6.84	7.07
Moisture Content %	42.55	43.91	45.26	48.44
Number of Blows	35	27	21	12
Liquid Limit Test			LL= 35.1%	
Test No.	1	2	3	4
Container No.	A	B	C	D
Wet soil & Container g	33.05	32.56	33.76	32.80
Dry Soil & Container g	28.60	28.10	28.70	27.60
Container Empty g	14.40	14.20	13.60	13.00
Dry Soil g	14.20	13.90	15.10	14.60
Loss of Water g	4.43	4.46	5.05	5.20
Moisture Content %	31.16	32.10	33.54	35.51
Number of Blows	42	33	23	14

The value for CBR for the studied soil ranges from 8% to 45% (Fig.7). The Federal Ministry of Works and (1997) specified a minimum value of 10% and 15% for soaked and unsoaked CBR for a sub-grade soil compacted at in OMC and MDD using BS proctor compaction method. The federal Ministry of Works and Housing (1974) also specified a minimum value of 30% for soaked CBR sub-base material compacted at its MDD and OMC using West Africa level of compaction. However, Adeyemi (1990) established a mathematical relationship of the form.

Y=1.04X +117.16 between unconfined compressive strength and unsoaked CBR. Compacted at the modified AASHTO level where Y = unconfined compressive strength and X = unsorted CBR. This gave values ranges from 121.kN/m^{2}to 125.39kN/m^2 for Y. These values are generally higher than that specified by De Graft Johnson and Bhatia (1969) when they reported that the Central Road Research Institute of India recommended cured unconfined compressive strength of 103.4kg/m^3 for road soil.

The soil/rocks below the highway pavement is expected to possess sufficient strength to support the structure or wheel load imposed on it. It must not swell or shrink excessively and must have proper permeability and drainage characteristics. Unfortunately, due to the heterogeneous nature of

the tropical soil and subsurface geological structures, the above conditions are rarely met and hence the strength of the sub-grade decreases and eventually, the pavement on it fails.
Most of the road failures were distributed while very few of them were localized which may be due to rock contact as reported by (Meshida, 1980). The road failure at Ogbagi section of the area is due to poor drainage system and poor construction of the road.

Table 4. The plastic limit result

Plastic Limit (PL) Test		
Test No.	1	2
Container No.	A	B
Wet Soil & Container g	27.11	29.69
Dry Soil & Container g	24.40	26.70
Container Empty g	13.20	14.40
Dry Soil g	11.20	12.30
Loss of Water g	2.71	2.99
Moisture Content %	24.20	24.30
Plastic Limit. PL =	24.3 %	
SAMPLE 2		
Test No.	1	2
Container No.	A	B
Wet Soil & Container g	26.48	28.96
Dry Soil & Container g	24.40	26.70
Container Empty g	13.20	14.40
Dry Soil g	11.20	12.30
Loss Water g	2.08	2.26
Moisture Content %	18.60	18.40
Plastic Limit PL =	18.5 %	

Electrical resistivity survey

The Pseudo section results show that there is highly resistive bedrock close to the surface at about 5 to 6 meter. It has a thin layer top soil follow by the hard rock. There are resistivity pathways in some section of the profile indicating sub surface fractures which can serve as an aquifer or change in rock type and properties. The FDM inversion shows the engineering properties.

In the first traverse, the resistivity value of the pseudo section range from 14656 to 1418 Ohm-m at depth of 2.5m between stations 1 to station 9. From station 9 to station 13, the resistivity are 833, 410, 261 and 200 at the same depth until it become very low at station 14 to station 18 at as low as 65.7, 37.2, 15.9 and 22.0 at the same depth of 3m which indicate the presence of highly conductive materials which may be water or metallic substance. Generally at depth of 5.0m the resistivity value is high except at station 16 where it is 343 Ohm-m. Below depth 5.0m from station 5 to station 18, the resistivity is extremely high indicating highly resistive bedrock.

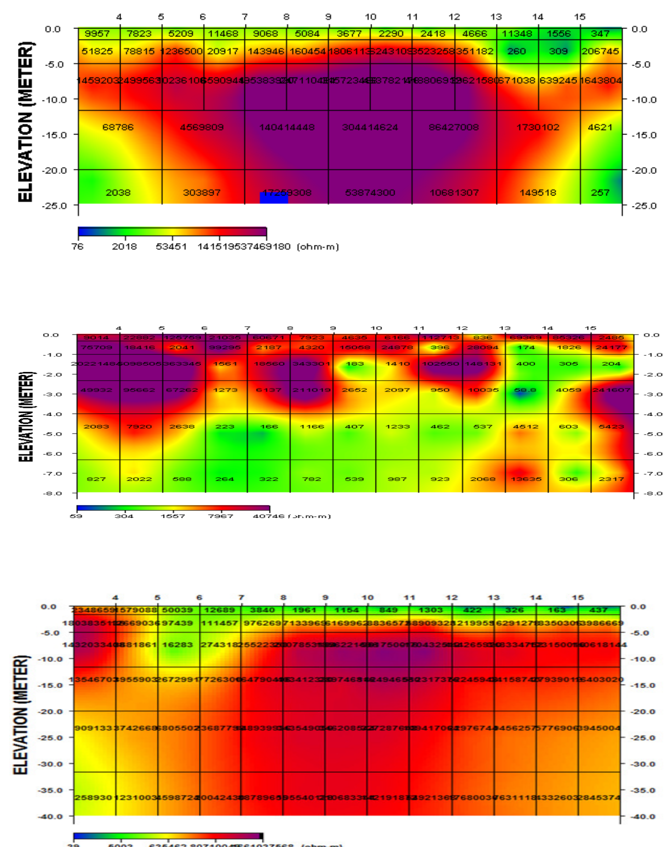

Fig. 8. The raw and modeled resistivity section along ERT for profile 1-3.

In the second traverse, the resistivity value at station 1 to station 5 is extremely high (> 6000 Ohm-m) from the surface to the depth of 2.0m. It is relatively high between station 6 and station 12 at depth of

1.0m. From elevation of -2.0m to -5.0m the resistivity value range from 4144 Ohm-m to 712 Ohm-m. At station 14 to station 16, the resistivity value is 400 Ohm-m, 160 Ohm-m and 51.3 Ohm-m respectively down to the depth of 4.0m indicating a conductive pathway.

In the third traverse, the resistivity value is very high (>10891 Ohm-m) at station 1 to station 6 down to and depth of 25.0m. At station 7 and 8 the resistivity value is 2363 Ohm-m and 822 Ohm-m at depth of 3.0m. At the same elevation of -3.0m the resistivity of station 9 to station 17 are 647 Ohm-m, 302 Ohm-m, 190 Ohm-m, 151 Ohm-m, 49.9 Ohm-m, 37.2 Ohm-m, 4.93 Ohm-m and 377 Ohm-m respectively indicative of weathered layer or a highly conductive material. Below the depth of 5.0m from station 10 to station 18, the resistivity is very high (>22091 Ohm-m) indicating a highly resistive hard bedrock

The 2-D dipole-dipole resistivity tomography image (Fig.8) display a generally thin low resistivity overburden with isolated near vertical high resistivity features (which may be a fracture zone) with significant depth extent of up to 5.0m or even greater in some places. There is also presence of linear features such as fractured zones or buried channel with surface expressions at distances 10-25m.

Conclusions

The natural moisture content of the soil range from 18.56% to 11.3%, specific gravity range from 2.683 to 2.761, consistency limit range from 44.8% to 33.1%, plasticity index range from 14.6% to 20.55%, linear shrinkage range from 9.6% to 11.0%, sand range 40% to 52% fines range 30.4% to 58.1%, clay range 37.4% to 23.7%.

The road pavement is constructed on laterite sub grade soil with mean low resistivity and thickness of 1486 ohm m and 2.5m respectively. The underlying weathered layer has a mean resistivity value of 250ohm-m and average thickness of 6.1m. The composition is typical of sandy clay/ clayey sand. The 2-D dipole-dipole tomography image confirms the relatively homogeneous nature of the subsurface and the uneven bedrock interface. The presumably fresh bedrock forms the third layer with very high resistivity in most places. The depth to the bedrock ranges between 4.8 and 10.1 m. The basement bedrock topography is uneven. The 2-D resistivity tomography image also shows that the overburden is generally thin (<5m) in most places. It however identifies isolated near vertical low resistivity features having depth extent of up to 20m typical of lineaments/fractures and buried channels with surface expressions at 0 to 25m. This study identifies the possible causes of the highway pavement failure in a typical basement complex area to include the following: Clayey topsoil/sub-grade soils have tendency of absorbing water which makes them swell and collapse under imposed wheel load stress which subsequently lead to road failure.

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